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PRESTRESSING OF REINFORCING BARS IN CONCRETE SLABS DUE TO CONCRETE EXPANSION INDUCED BY ALKALI-SILICA REACTION

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ABSTRACT

Alkali-silica reactions (ASR) in concrete bridges have been a major concern worldwide for many decades. In Denmark, several bridges are severely damaged due to ASR and over 600 bridges have the potential to develop ASR in the future. The majority of these bridges are slab-bridges. Despite the many cases, experimental research on structural safety and residual load carrying capacity of ASR-damaged bridges is limited. As ASR causes severe cracks in the concrete, which may affect the concrete compressive and tensile strength, concerns have been directed towards the residual shear capacity. Yet, against all expectations, existing experimental studies have shown that the residual shear capacity of ASR deteriorated specimens is not significantly reduced compared to similar undamaged specimens - even in cases with severe ASR-induced crack formation. These results are often explained by a favourable prestressing effect in the reinforcement induced by ASR expansion. In order to develop a general shear model, the ASR-effects need to be taken into account. Unfortunately, the number of studies focusing on the actual quantification of this prestressing effect is limited. Additionally, the studies are mostly based on relatively small laboratory specimens or beams and structural elements with shear reinforcement. This paper provides the first plausible explanation on why these results are unrepresentative for slab elements. Additionally, this paper quantifies the prestressing effect by analysing full-scale slabs, obtained by severe ASR-damaged bridges located in Denmark. The results show that in the early stage, the ASR-induced prestressing effect develops simultaneously with the ASR-induced cracks. Subsequently the cracks develop further, while the prestressing effect reaches a maximum. The results and existing studies are used to discuss an existing assessment method.

Keywords: ASR, restraint expansion, slabs without shear reinforcement, existing bridges, residual shear capacity.

1. Introduction

Alkali-silica reactions (ASR) in concrete bridges have been a major concern worldwide for many decades (Charlwood, 1994) (Wigum, 2006), for example in South Africa (Blight et al., 1981). In Denmark, several bridges are severely damaged due to ASR and more than 600 bridges have the potential to develop these damages (Larsen, 2013).

As ASR causes severe cracks in the concrete, it may affect the concrete compressive and tensile strength. Therefore, it is feared that the residual shear capacity may be affected too. This especially holds in the case of concrete elements *without* shear reinforcement (e.g. slabs), since - in contrast to elements *with* shear reinforcement - the shear force is carried by the concrete itself only. Since the majority of the (potentially) ASR-damaged bridges in Denmark are slab bridges, concerns have been directed towards the residual shear capacity concrete slabs. Similar concerns for slabs have been raised in the Netherlands by ((Bakker, 1999) published in (Bakker, 2008)).

Currently, no commonly accepted method exists to account for the mentioned ASR-effects in the residual load carrying capacity of concrete slab elements. As a consequence engineers are often forced to make rough estimates. Existing codes and standards are mainly developed as design tools, which are intended for the design of new structures rather than for the assessment of (ASR-) damaged structures. Nonetheless,

methods that account for ASR in structural calculations do exist. The CUR-recommendation provides inspection and assessment rules for concrete structures in which the presence of ASR is suspected or has been established (CUR, 2008). Cope et al. (1994) and Jones and Clark (1998) have worked on numerical models that account for ASR. However, the CUR-recommendations are rather conservative (Bakker, 2008) and the numerical methods do not focus on the structural behaviour of concrete slabs. Therefore, there is a need for a general shear model that takes ASR-effects into account, which is applicable beyond its empirical basis.

Experimental research within the field of residual shear capacity of ASR deteriorated slab elements has already been conducted. Against all expectations, these studies have shown that the residual shear capacity is not reduced as much as anticipated from the drastic reductions of the concrete compressive strength - even in cases with severe ASR-induced crack formation (den Uijl and Kaptijn, 2003) and (Schmidt et al., 2014). Similar results have been found for beams without shear reinforcement (ISE, 1992), ((Company, 1986) published in (Clark, 1989)), (Vejdirektoratet, 1986), (The Road Directorate, 1990), (Cope and Slade, 1992), (Chana and G. A. Korobokis, 1991) and (Ahmed et al., 1998). In the literature these results are often explained by a favourable prestressing effect in the longitudinal reinforcement induced by ASR expansion (Fujii et al., 1986), (ISE, 1992), ((Company, 1986) published in (Clark, 1989)), (The Road Directorate, 1990), (Cope and Slade, 1992), (Chana and G. A. Korobokis, 1991), (Ahmed et al., 1998), (den Uijl and Kaptijn, 2003) and (Hansen et al., 2016). This thesis is supported by test where beams without shear reinforcement were stiffer than non-ASR-damaged beams (published in (Clark, 1989).

In order to develop the model more qualitative and quantitative knowledge on the prestressing effect is required. Much relevant research has already been published within this field. Among others, Cope et al. (1994) studied the development of ASR-induced expansion and its effects on the concrete properties and the structural behaviour. Multon and Toutlemonde (2006), Berra et al. and Kagimoto et al. (2014) studied the effects of restraining conditions on the ASR-induced expansion. Inoue et al. (1989) have shown that only 40% of the ASR induced prestress is left after two years of drying (published in (ISE, 1992)). Unfortunately, the majority of existing studies are not directly representative for slab elements since they are based on relatively small laboratory specimens or beams and structural elements with shear reinforcement, in which ASR has been accelerated. The number of studies focusing on the actual quantification of this prestressing effect in slabs is limited.

This paper provides the first plausible explanation on why most of these existing quantitative studies results are unrepresentative for slab elements. Additionally, this paper quantifies the prestressing effect by analysing full-scale slabs, obtained by severe ASR-damaged bridges located in Denmark. Finally, the representativeness of existing experiments and the applicability of existing method for assessing the prestressing effect are discussed.

- Section 2, *ASR induced expansion in 2D reinforced structures*, gives a plausible explanation of the development of the crack formation and ASR-induced prestressing in a slab without shear reinforcement.
- Section 3, *Most of existing experiments are not representative for slab elements*, explains why existing test results from small laboratory specimens or beams and structural elements with shear reinforcement are unrepresentative for slab elements.
- Section 4, *Materials and Existing methods*, describes the tested bridges and the test method.
- Section 5, *Results*, presents the results of the strain measurements.
- Section 6, *Discussion*, discuss the implications of quantification, the representativeness of existing experiments and the applicability of existing method for assessing the prestressing effect.
- Section 7 draws the conclusions.

To understand why the ASR-induced prestressing effect for slabs differs from prestressing effect for small laboratory material samples and beams or structures with shear reinforcement it is important to know how ASR expansion and cracks develop in a reinforced structure.

2. ASR-induced expansion in 2D reinforced structures

Figure 2 shows the development of the ASR-induced prestressing and crack formation in a slab without shear reinforcement, which is based on a composition of published research (Hobbs, 1988), (ISE, 1992), (Multon and Toutlemonde, 2006). The considered slab is isotropic reinforced (i.e. same reinforcement ratio in x - and y -directions and identical at the top and bottom face) with a sufficiently anchored orthogonal mesh. The slab is not subjected to external actions. The figure shows the following phases of ASR progression:

a) The ASR expansion occurs in or around the reactive silica particles (Hobbs, 1988). The expansion may develop equally in all directions, since the restraining stiffness of the surrounding uncracked cement paste is approximately isotropic.

b) The internal pressure due to the ASR expansion makes the surrounding cement paste crack and multidirectional micro cracks occur locally around the reactive particle.

c) Due to the micro cracks the reinforcement will be stressed to tension (prestressed) and thus induce compressive stresses in the concrete.

d) The axial (x - and y -direction) stiffness of the slab cross sections will remain unaffected due to the prestressing. Contrarily, the vertical (z -direction) stiffness will decrease during the crack formation. Due to the anisotropic stiffness (softest in the z -direction) and the axial concrete stresses further ASR expansion and crack opening will develop mostly in the vertical direction (ISE, 1992). Multon and Toutlemonde (2006) call this “expansion transfer”.

e) Subsequently micro cracks will grow into macro cracks with coarser cracks, parallel to the direction of the reinforcement and the compressive stresses.

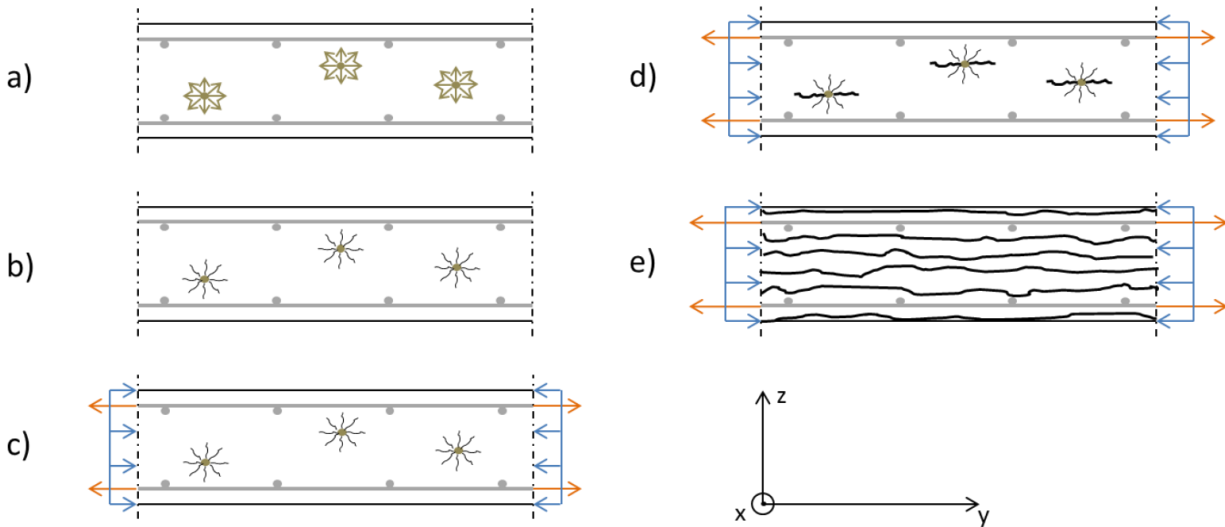


Figure 2, phases of ASR progression in isotropic reinforcement slab element. The sections are also representative for sections normal to the y -axis.

3. Most of existing experiments are not representative for slab elements

As mentioned above, the majority of previous studies on ASR-induced expansion are not directly representative for slab elements. They are based on small specimens or on specimens where either the restraining conditions or the exposure conditions have induced unrepresentative crack formation.

In experiments conducted without restraints, the cracks form with random orientation as the ASR-expansion has no preferred direction. Figure 3 (left) shows three concrete specimens with ASR cracks. Specimen 1 has no restraints where specimens 2 and 3 have embedded longitudinal reinforcement only. The figure shows, that the crack formation for specimen 1 is more random than the crack formation for

specimens 2 and 3. The crack formation for unrestraint specimens correspond to the early phases of ASR progression for isotropic reinforced slab elements, Figure 2 (b).

Some experiments are conducted with *1D*, *2D* and *3D* restraints to better simulate the restraining conditions in real structures, e.g. reinforcement or adjacent structures. In specimens with *1D* restraints the structure has two directions with softer stiffness. As a result the cracks form with random orientation in a section perpendicular to the restrained direction. Figure 3 (centre) shows an example of a *1D*-restrained specimen by embedded reinforcement bars. The cracks are randomly orientated in the cross section. The crack formation in specimens with *2D* restraints is similar to slab elements as there is one preferred direction for expansion. In specimens with *3D* restraints (e.g. shear reinforcement) the cracks form with random orientation as the ASR-expansion has no preferred direction. The influence of the restraining conditions on the expansion behaviour is also shown by experiments (Multon and Toutlemonde, 2006).

Besides unrepresentative restraining conditions, some experiments are conducted with unrepresentative exposure conditions. Typically, ASR is accelerated by exposing all surfaces with water (and alkalis), contrary to bridge decks, which typically are exposed on the top face only. The four-sided exposure induces a fast ASR-expansion in the cover on all surfaces. Figure 3 (right) shows a cross section of a beam, which has been exposed on all surfaces during the acceleration period. It shows that the cracks are formed parallel to the surfaces and only in the cover, unlike the crack formation in slab elements. Hobbs (1988) did also conclude that this crack formation “*is uncharacteristic of the cracking induced by alkalis inherent in the concrete*”.

Furthermore, it is not clear whether results from experiments with small-scale specimens are representative for full-scale structures. More researchers have concluded that the size of the test specimens affect the ASR-induced expansion, e.g. (Jones and Clark, 1996).

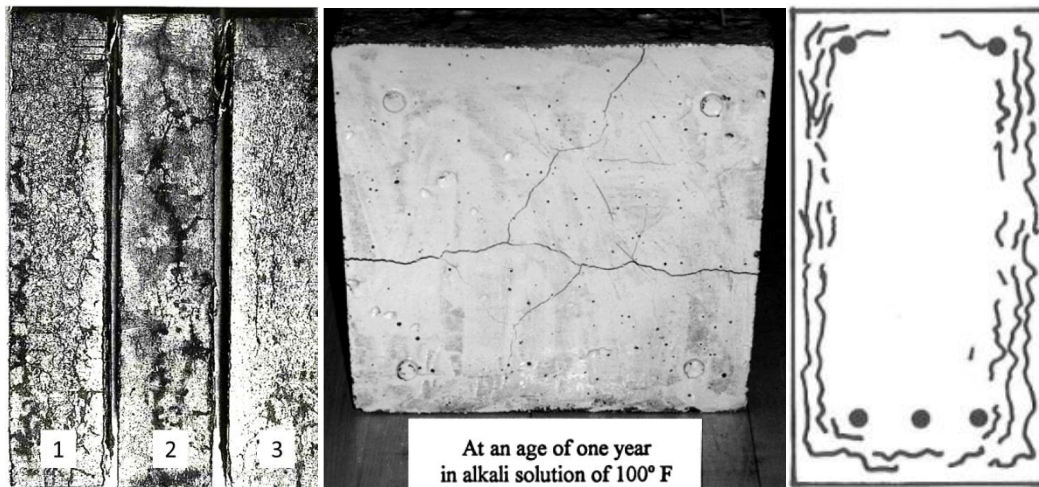


Figure 3, (left) ASR-induced crack in an unrestraint specimen (specimen 1) and in specimens restrained by a reinforcement degree on 1% (specimen 2) and 4% (specimen 2), adapted from (Hobbs, 1988) – (centre) ASR-induced cracks in a cross section of a specimen with *1D* restraints, from (Fan and Hanson, 1998) – (right) ASR-induced cracks for a beam without shear reinforcement, strongly exposed on all surfaces, adapted from (Vejdirektoratet, 1986)

4. Materials and Methods

In Denmark the most bridges suffering from ASR-deteriorations are built with fast reactive aggregates typically in the sand fraction, only (Wigum, 2006).

4.1 The bridge Gammelrand over Skovvejen

Gammelrand over Skovvejen was a three span concrete bridge which was built in 1976. The total bridge length was 35.7 m, the width was 9.1 m and the height of the bridge deck was 0.73 m. The entire bridge deck suffered from severe ASR-deteriorations. Due to uncertainties about the structural safety it was decided to replace the bridge and it was demolished in 2010. In connection with the demolition, four 7.65

m long beams were sawed and stored. In 2013 the ASR-induced tensile strain in the longitudinal reinforcement was measured in the top reinforcement bars in two of the beams. The position of the measuring points was chosen with respect for anchoring conditions. Generally, the reinforcement in the beams was not identical. Table 1 shows the amount of reinforcement in both beams.

Table 1. Registered longitudinal reinforcement for beam 2 and 4

Beam number – section	Top reinforcement	Bottom Reinforcement
2-1	1 Ø20 + 2 Ø25	3 Ø25
2-2	1 Ø20	3 Ø25
4-1	2 Ø20	3 Ø25
4-2	2 Ø20 + 1 Ø25	3 Ø25

4.2 The bridge *Lindenberg Pæledæk*

Lindenberg Pæledæk is a pile supported concrete slab bridge which is built in 1966-1967. The bridge consists of a southern and a northern part. The total length of the northern bridge is 312 m and the total length of the southern bridge is 120 m. The width of the bridge is 10.0 m. The height of the deck is 0.3 m. Figure 4 shows a photo from beneath the deck and an excerpt of the *As Build* drawing for a cross section. Based on this drawing, the main longitudinal reinforcement in the bridge deck consists of Ø16mm bars per 320 mm in the top and Ø16mm bars per 160 mm in the bottom. Inspections of the bridge have shown severe ASR damages in the bridge deck and columns. Due to uncertainties about the shear capacity of the bridge deck it was decided to test the shear capacity of beams cut from the deck in 2014. Six slabs were cut from the northern bridge deck. Each slab was cut into seven beams; three beams for testing the shear capacity and four 200 mm beams, named A-D, for measuring the strain in the reinforcement and to test the material properties of the damaged concrete.

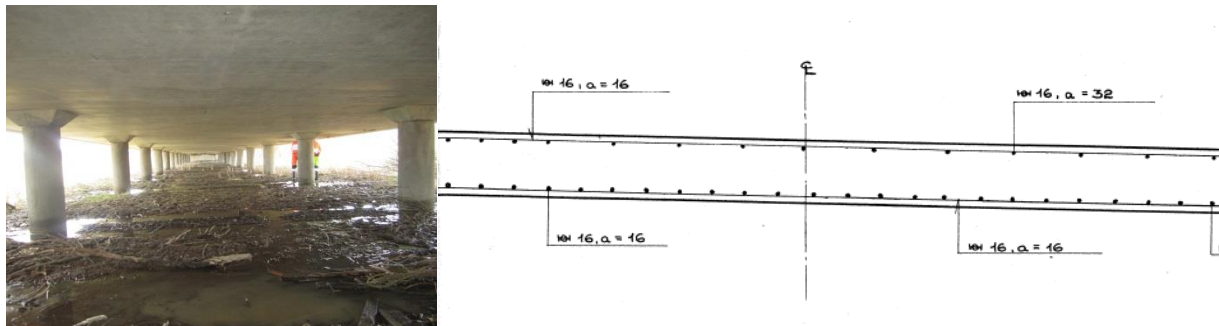


Figure 4, (left) photo of deck and columns from *Lindenberg Pæledæk* – (right) reinforcement in the bridge deck from the *As Build* drawing

4.3 Method - Strain measurements

In order to quantify the ASR-induced prestressing of the longitudinal reinforcement, measurements were performed on beams cut from the two bridges. The measurements were conducted by mounting strain gauges on the reinforcement bars and cutting the rebars free from the concrete, see Figure 5. Before mounting the strain gauge; the concrete was locally removed around the reinforcement bars and the bare part of the rebar was grinded and cleaned. The reinforcement bar was cut about 10 cm from the strain gauge. The strain measurements on all rebars in each section were started simultaneously.

The calculations in this paper are based on the average of measured Young's moduli for the reinforcement steel, corresponding to 186.5 GPa for *Gammelrand over Skovvejen* and 214.0 GPa for *Lindenberg Pæledæk*.



Figure 5, (left) photo of a strain gauge mounted on a reinforcement bars – (right) photo of a cut reinforcement bar

5. Results

5.1 The bridge Gammelrand over Skovvejen

Figure 6 shows the measured tensile strain in the reinforcement during the cutting. A few measurements were clearly affected by insufficient anchoring conditions. Only the strain measurements with good anchoring condition are shown.

For beam 2; rebar 1, 2 and 4 were cut in one end after approx. 2 minutes, which is the reason of the large, almost instantaneously, increase in the strain measurement. Afterwards, the rebars were loosened from the surrounding concrete, which is the reason of the small fluctuations within the first 20 minutes. Rebar 3 was cut in one end after approx. 3 minutes and loosened from the surrounding concrete after 8-20 minutes. The figure shows that the tensile strains in beam 2 varied from approximately 1.2‰ to 1.4‰ for the Ø20mm rebar, corresponding to tensile stresses of 224 to 261 MPa. The tensile strain in the Ø25mm rebar was 0.7‰, corresponding to tensile stresses of 131 MPa.

For beam 4; rebars 1-4 were cut after approx. 200, 75, 50 and 100 minutes, respectively. The figure shows that the tensile strain was approx. 1.25‰ for Ø20mm rebar, corresponding to tensile stresses of 233 MPa. The tensile strain in the Ø25mm rebar was approx. 0.9‰, corresponding to tensile stresses of 168 MPa.

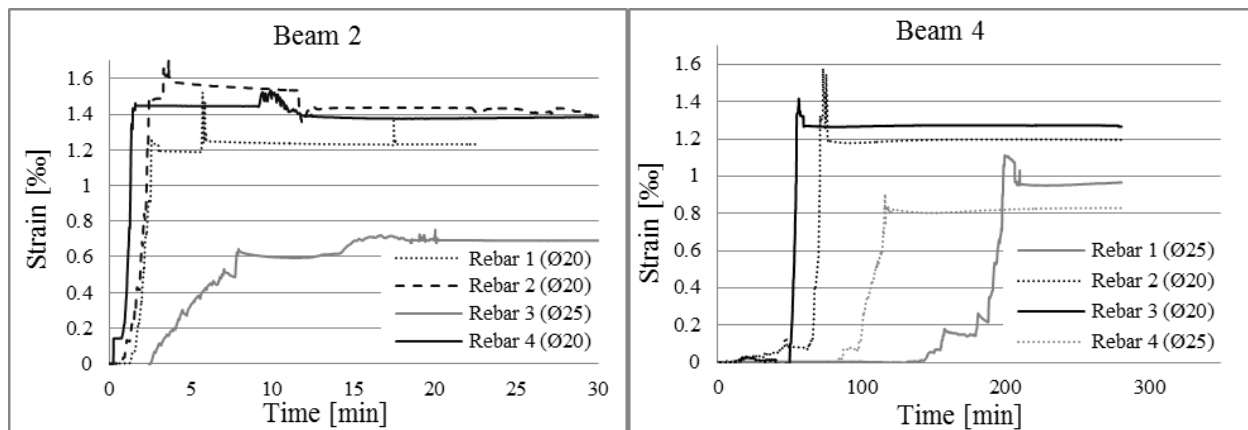


Figure 6, the measured tensile strain in the reinforcement in *Gammelrand over Skovvejen*

5.2 The bridge Lindenberg Pæledæk

Figure 7 shows the measured tensile strain for the rebars in the six slabs. Furthermore, the figure shows photos of the slabs wherein the strain was measured. The measurements were conducted after the slabs were cut into beams. Few measurements were clearly affected by insufficient anchoring conditions. Only the strain measurements with good anchoring condition are shown.

The measurements show:

- Slab 1; the tensile strains varied from approximately 0.7‰ to 1.1‰, corresponding to stresses of 150 to 235 MPa. The same variation is also seen within beam A, only. The photo shows severe ASR cracks and corrosion.
- Slab 2; the tensile strains were approximately 1.0‰, corresponding to stresses of 214 MPa. The variation of the measurements was practically zero. Only one measurement per rebar was conducted. The photo shows only few small ASR-induced cracks.
- Slab 3; the tensile strains were 1.3‰ and 0.7‰, corresponding to stresses of 150 to 278 MPa. Only one measurement per rebar was conducted. The photo shows ASR cracks.
- Slab 4; the tensile strains varied from approximately 0.9‰ to 1.3‰, corresponding to stresses of 193 to 278 MPa. Only one measurement per rebar was conducted. The photo shows severe ASR cracks and corrosion.
- Slab 5; the tensile strains varied from approximately 1.1‰ to 1.4‰, corresponding to stresses of 235 to 300 MPa. Only one measurement per rebar was conducted. The photo shows ASR cracks.
- Slab 6; the tensile strains varied from approximately 1.1‰ to 1.4‰, corresponding to stresses of 235 to 300 MPa. Two measurements were conducted in beam A; they show practically the same tensile strain. The photo shows severe ASR crack, especially in the top of the section (left side of the photo).

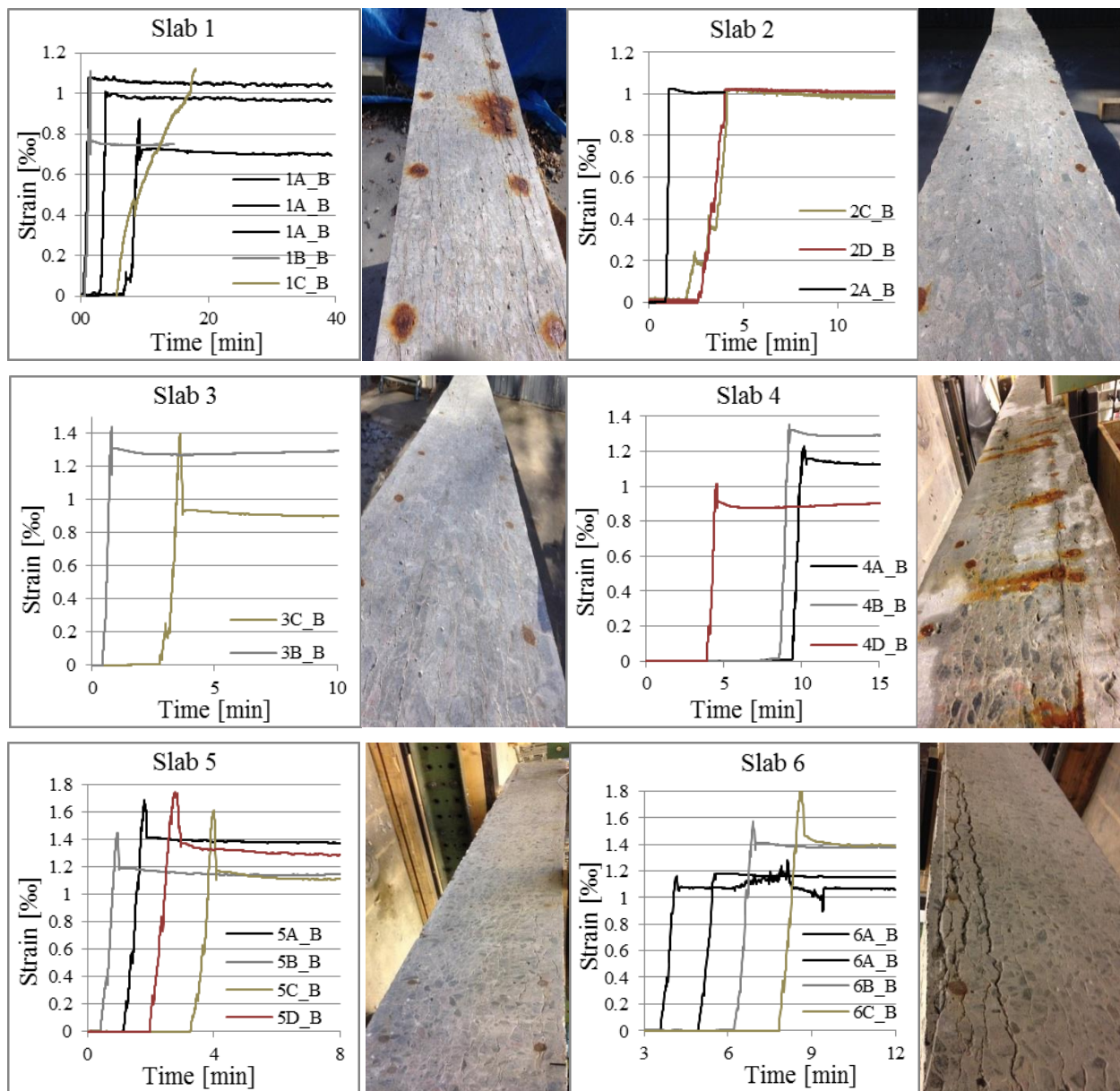


Figure 7, the measured tensile strain in the reinforcement and photos of the slab sections. The measurements are named after the slab and beam were they are conducted, e.g. 2D_B is a measurement conducted on slab 2, beam D on bottom reinforcement (_B).

6. Discussions

This paper has presented:

- A plausible explanation on why most of the existing studies within the field of ASR expansion are unrepresentative for slab elements.
- A quantification of the ASR-induced prestressing effect by analysing two ASR-deteriorated bridges in Denmark.

Gammelrand over Skovvejen

The measurements from *Gammelrand over Skovvejen* shows that the tensile strains in the Ø20mm rebars were approximately at the same level, corresponding to approx. 260 MPa. This result is expected since the beams were placed close to each other in the bridge and may have the same degree of ASR deterioration based on visual appearance of the cracks. The measured tensile strain in the Ø25mm rebars is significantly less, corresponding to approx. 160 MPa and shows a larger deviation. The lower strain level in the

Ø25mm rebars may be explained by a larger reinforcement degree. Existing experiments have shown that a certain concrete compressive stress can resist further ASR expansion (ISE, 1992), (Multon and Toutlemonde, 2006), (Berra et al., 2010), (Kagimoto et al., 2014). Due to large uncertainties about the reinforcement in the tested beams it has not been possible to determine the level of concrete compressive stresses.

Lindenberg Pæledæk

The measurements of the tensile strain from *Lindenberg Pæledæk* show that the prestressing effect in the reinforcement is not proportional to the visual appearance of ASR-induced cracks. It appears clearly by comparing the measurements and photos from slab 1 and 3. The cracks in slab 3 are significantly coarser but the measured tensile strains are approximately equal. This indicates that the prestressing effect develops proportional to the ASR crack formation up until a certain level, in this structure about 1.0‰, whereafter the prestressing effect remains constant and only the ASR crack formation progresses. Similar conclusions can be drawn from existing small-scale experiments with 2D restrained specimens (Multon and Toutlemonde, 2006). Also this behaviour can be explained by the equilibrating concrete compressive stresses. The calculated stresses are 1.5 and 2.2 MPa in the centre and in the bottom of the section, respectively, by assuming; all reinforcement bars (top and bottom) were prestressed 1‰ and a straight concrete stress distribution. These stresses are in the same level as the maximum stresses (1-2,5 MPa) in the above-mentioned. Their studies have showed that creep can be the reason why the prestressing only increases up until a certain level.

The measurements show furthermore a tendency to a larger deviation of the tensile strains in slabs with courser ASR cracks. This may be due to insufficient anchoring. Since the measurements were conducted after the slabs were cut from the bridge, the anchoring condition is expected to be more vulnerable to horizontal cracks.

More knowledge about the development of the prestressing effect is need, e.g. on how the reinforcement ratio affect the development.

Measurements and Existing methods

The cutting into beams may have had an influence on the reinforcement strains since the reinforcement ratio may differ from the original one in the bridge. In case of a smaller reinforcement ratio, an instantaneous increase in the reinforcement strain may occur due to a reduction of the stiffness of the restraint.

Measuring the prestressing directly on the reinforcement bars is inconvenient since it need to be cut and replaced. Therefore, many publications recommend *Crack Width Summation* (Jones and Clark, 1994) as a method to quantify the prestressing effect in a given direction. For this, the method uses accumulating crack widths in the given direction. Unfortunately, the method cannot be used for slab bridges. The cracks on the bottom face of slabs are not vertical cracks that also intersect the reinforcement, which is assumed in the method. The cracks are presumably the uneven horizontal cracks near the surface, which intersect, see figure 2 (e). The inapplicability of the method for slabs is further confirmed by the measurements from *Lindenberg Pæledæk*:

- The ASR cracks progress after the prestressing effect has reached its maximum constant level.
- A significant prestressing effect was measured in slabs with only minor cracks, see slab 2 in Figure 7.

To develop an applicable and reliable non-destructive assessment method for prestressing effect for slabs more research are needed.

Implications

On the basis of the results discussed in this paper, a (research) hypothesis for the development of shear capacity in ASR-damaged slab bridges can be formulated. Figure 8 schematically shows how three relevant structural properties most likely will evolve over time. The figure shows three time steps, *No ASR*, *ASR* and *After ASR* on the horizontal axis. The vertical axis shows the development of *Residual*

shear capacity (black line), Concrete strength (grey line) and ASR-induced prestressing effect (red line). In the time step *No ASR* it is well-known that the concrete strength increases over time. Since the prestressing effect is unaffected the *Residual shear capacity* will increase slightly. In the time step *ASR*, the *Prestressing effect* will increase in the beginning due to the ASR expansion, as shown in the experiments. Existing experiments on unrestrained specimens have shown that the *Concrete strength* decreases as function of the ASR-induced expansion, e.g. (Cope et al., 1994). Due to the large increase in the prestressing effect the *Residual shear capacity* increases. When the *Prestressing effect* reaches its maximum constant level, the *Residual Shear Capacity* will decrease slightly due to the decreasing *Concrete strength*. In the time step *After ASR* it is assumed that the structure has been repaired so that the ASR process has stopped, e.g. due to a new waterproofing membrane, which then results in drying of the concrete. Experiments have shown that only 40% of the ASR induced prestress is left after two years of drying ((Inoue et al., 1989) published in (ISE, 1992)). Therefore the *Residual shear capacity* decreases.

The time step *No ASR* is well documented. The time step *ASR* is based on only few reliable experiments and other experiments which are not necessarily representative for slab elements. The time step *After ASR* is based on one old experiment. Since the reliability of the development in this time step is low, the curves are dashed.

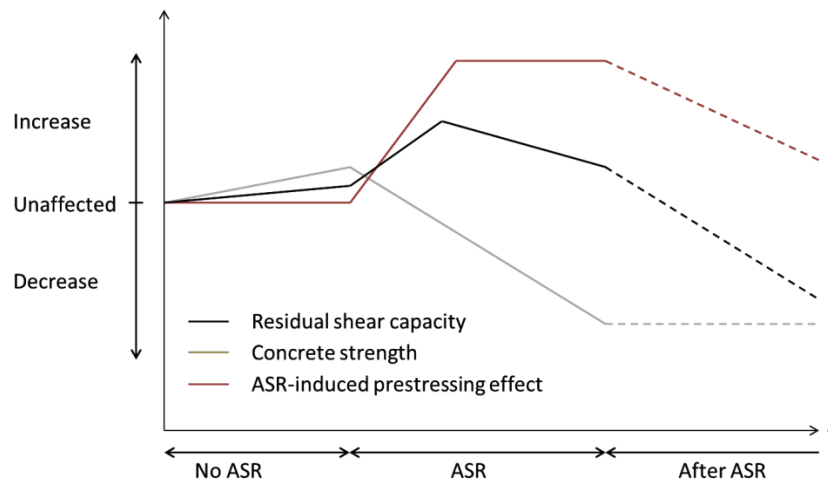


Figure 8, how ASR deterioration in slabs affects the prestressing effect (red line), the concrete strength (grey line) and the shear capacity (black line) before, during and after ASR.

Tests of ASR deteriorated structural elements have shown that the residual shear capacity was not or only slightly reduced. However, the results of published experiments are only a snapshot in time. More knowledge about how shear capacity develops as function of ASR deterioration is needed to decide the right future for our structures.

7. Conclusions

The ASR-induced tensile strain in the reinforcement bars in bridge decks has been experimentally investigated. The results show:

- That it was possible to measure a considerable prestressing in the reinforcement, from 140 MPa to 280 MPa.
- That the tensile stresses in Ø20mm rebars are significant larger than the stresses in the Ø25mm rebars.
- That the prestressing effect develops proportional to the ASR crack formation up until a certain level whereafter the prestressing effect remains constant and only the ASR crack formation progresses.
- Good agreement with existing experiments with 2D restrained small-scale specimens.
- *Crack Width Summation* is not applicable for slab elements.

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